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NATIONAL DAM SAFETY PROGRAM. BARCROFT DAM (VA 05901), POTOMAC R--ETC(U)

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POTOMAC RIVER BASIN

Name Of Dam: BARCROFT DAM
Location: FAIRFAX COUNTY, VIRGINIA
Inventory Number: VA 05901

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

PREPARED BY
NORFOLK DISTRICT CORPS OF ENGINEERS
803 FRONT STREET
NORFOLK, VIRGINIA 23510

MAY 1978

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER VA 05901	2. GOVT ACCESSION NO.	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) Phase I Inspection Report National Dam Safety Program Barcroft Dam Fairfax County, Virginia		5. TYPE OF REPORT & PERIOD COVERED Final 6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) U. S. Army Corps of Engineers, Norfolk District- 10 H. E. Strawsnyder		8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS 9 Final rept.		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS U. S. Army Engineering District, Norfolk 803 Front Street Norfolk, VA 23510		11 12. ~~REPORT DATE~~ May 1978 13. NUMBER OF PAGES 37
14. MONITORING AGENCY NAME & ADDRESS (if different from Controlling Office)		15. SECURITY CLASS. (of this report) Unclassified 15a. DECLASSIFICATION/DOWNGRADING SCHEDULE
16. DISTRIBUTION STATEMENT (of this Report) 12 47p. Approved for public release; distribution unlimited.		
17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report) 6 National Dam Safety Program. Barcroft Dam (VA05901), Potomac River Basin, ~~Holmes Run~~, Fairfax County, Virginia. Phase I Inspection Report.		
18. SUPPLEMENTARY NOTES Copies are obtainable from National Technical Information Service, Springfield, Virginia 22151		
19. KEY WORDS (Continue on reverse side if necessary and identify by block number) Dams - VA National Dam Safety Program Phase I Dam Safety Dam Inspection		
20. ABSTRACT (Continue on reverse side if necessary and identify by block number) (See reverse side) 411 030		

DD FORM 1 JAN 73 1473 EDITION OF 1 NOV 65 IS OBSOLETE

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

20. Abstract

Pursuant to Public Law 92-367, Phase I Inspection Reports are prepared under guidance contained in the recommended guidelines for safety inspection of dams, published by the Office of Chief of Engineers, Washington, D. C. 20314. The purpose of a Phase I investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general conditions of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

Based upon the field conditions at the time of the field inspection and all available engineering data, the Phase I report addresses the hydraulic, hydrologic, geologic, geotechnic, and structural aspects of the dam. The engineering techniques employed give a reasonably accurate assessment of the conditions of the dam. It should be realized that certain engineering aspects cannot be fully analyzed during a Phase I inspection. Assessment and remedial measures in the report include the requirements of additional indepth study when necessary.

Phase I reports include project information of the dam and appurtenances, all existing engineering data, operational procedures, hydraulic/hydrologic data of the watershed, dam stability, visual inspection report and an assessment including required remedial measures.

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PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam: Barcroft Dam #VA 05901
State: Virginia
County: Fairfax County
USGS Quad Sheet: Annandale
Stream: Holmes Run

Barcroft Dam is a 69-foot high massive concrete gravity dam constructed in 1915. In June 1972, during Tropical Storm Agnes, a portion of the dam failed, draining the lake. The owners retained the services of Whitman, Requardt and Associates of Baltimore, Maryland as engineering consultants to restore the dam. This action has resulted in full restoration of the dam along with a program of periodic inspections. The renovated dam has a bascule gate mounted on the dam. This gate serves as a broad crested weir. The dam serves as a storage structure providing recreational impoundment known as Lake Barcroft. It is located on Holmes Run north of Virginia Highway 244 (Columbia Pike) one mile west of Bailey's Cross Roads, Fairfax County, Virginia. Barcroft Dam is owned by the Barcroft Lake Management Association and managed by the Lake Barcroft Watershed Improvement District.

The Corps criteria requires a spillway design flood equal to the Probable Maximum Flood (PMF). The project will not pass the PMF without overtopping the dam, and therefore, the spillway capacity is inadequate. The spillway is not considered seriously inadequate, as the project will pass more than one-half the PMF without overtopping the dam. A stability check based on the PMF indicates the design of the dam is within acceptable stability limits. Whitman, Requardt and Associates have developed an adequate and valid set of engineering data and they are retained by the owner to insure the adequacy of the dam. The visual inspection revealed no apparent problems. There are no immediate needs for remedial measures.

APPROVED

Newman A. Howard, Jr.
NEWMAN A. HOWARD, JR.
Colonel, Corps of Engineers
District Engineer

Date: 12 June 1978

BARCROFT DAM

SECTION 1 - PROJECT INFORMATION

1.1 General

1.1.1 Authority: Public Law 92-367, 8 August 1972, authorized the Secretary of the Army, through the Corps of Engineers to initiate a national program of safety inspections of dams throughout the United States. The Norfolk District has been assigned the responsibility of supervising the inspection of dams in the Commonwealth of Virginia.

1.1.2 Purpose of Inspection: The purpose is to conduct a Phase I inspection according to the "Recommended Guidelines for Safety Inspection of Dams" (Appendix F - Reference 1). The main responsibility is to expeditiously identify those dams which may be a potential hazard to human life or property.

1.2 Project Description

1.2.1 Dam and Appurtenances: Barcroft dam is a 69-foot high, cyclopean masonry gravity dam with a 52-foot wide base (Appendix A - Plates 2 through 4). The dam was constructed on a rock foundation with core walls extending into earthen embankments. The embankments tie into rock abutments for an approximate total dam length of 500 feet. The elevation of the top of the structure is 211.5 feet MSL USGS datum with the embankments gradually rising to over elevation 222 feet.

A Bascule gate is mounted on the main spillway portion of the dam, which is inclined 20° from the vertical with its crest at elevation 209.1 (Appendix B - Plate 1). The gate serves as a broad crested weir. It is 12 feet in height and 151 feet in length.

An intake tower and control building is located on the upstream face of the spillway section. The tower is a hollow concrete structure which allows a controlled flow of water to pass through the dam. At elevation 144 a 30-inch diameter pipe with gate valve runs through the tower to the downstream side of the dam. At approximate elevations 160, 180, and 195, 20-inch diameter pipes with gate valves allow lake water to enter the inside of the tower. A 24-inch diameter pipe allows this water to be discharged to the downstream side of the dam at approximate elevation 142.

Four removable steel gates with rubber seals are located on the crest of the "ogee" portion of the spillway just left of the intake tower. These gates are about 4.5 feet in height. The length of each of the two end gates is about 6 feet. The length of each of the two center gates is 7 feet. The gates are separated by concrete piers 8 inches thick. The overflow elevation of the gates is 208.3.

The structure does not have any warm water outlets, diversion tunnels, or ungated spillways.

1.2.2 Location: Barcroft Dam is located on Holmes Run north of Virginia Highway 244 (Columbia Pike) one mile west of Bailey's Cross Roads, Fairfax County, Virginia (Appendix A - Plate 1).

1.2.3 Size Classification: The dam is classified as an "intermediate" size structure because of its height (69.0 feet) and impoundment (3020 acre-feet), according to Section 2.1.1 of Reference 1.

1.2.4 Hazard Classification: The dam is located in an urban area and was therefore given a high hazard classification in accordance with guidelines contained in Section 2.1.2 of Reference 1, Appendix F. The hazard classification used to categorize dams is a function of location only and has nothing to do with its stability or probability of failure.

1.2.5 Ownership: Barcroft dam is owned by the Barcroft Lake Management Association (BARLAMA) and managed by the Lake Barcroft Watershed Improvement District (LBWID). The district is governed by the Northern Virginia Soil and Water Conservation District, Commonwealth of Virginia. A complete breakdown of the ownership and management is provided in Appendix E.

1.2.6 Purpose of Dam: Barcroft Dam serves as a storage structure providing a recreational impoundment known as Lake Barcroft. The dam serves no other purposes.

1.2.7 Design and Construction History: The dam was originally owned by the Alexandria Water Company (AWC) and served as a storage structure for municipal water supply. Construction was initiated in 1913 and completed and put into service by 1915. The structure was essentially the same as described in Section 1.2.1 with one main exception. It had an overflow "ogee" spillway, at an elevation of 204 feet, instead of the Bascule gate. The working capacity of the lake at that time was set at elevation 210.

In 1942 AWC added 24 manually operated gates to the overflow spillway. Along the remaining portion of the dam, an 18-inch high parapet wall was built to bring the water capacity up to elevation 211.5.

In 1950 AWC abandoned the structure and sold it to private interests who developed the lake into the Lake Barcroft Community. Eventually, a Lake Barcroft Community Association, comprised of home owners within the development, was formed. By 1970, they created BARLAMA and purchased the dam, lake and associated property.

During Tropical Storm Agnes, in June 1972, the dam failed. The right embankment was breached, draining most of the lake and destroying its recreational services. BARLAMA retained the services of Whitman, Requardt and Associates of Baltimore, Maryland to provide a Study of Long Range Improvements (Appendix F.- Reference 2). The LBWID was created as an avenue to finance the improvements and to manage the surrounding watershed. Improvements were implemented and completed by 1974 and mainly consisted of replacing the breached embankment, modifying the intake tower, replacing the manually operated gates with the Bascule gate, and adding downstream slope protection. In 1977, work was performed to remedy erosion of portions of the slope protection.

1.2.8 Normal Operational Procedures: The Bascule gate is designed to maintain a constant water level automatically at elevation 208.5. Any rise in the water level causes the gate to gradually open. When fully open, the Bascule gate is at elevation 197.1 feet and it serves as a spillway. The gate will automatically begin to close when the water level recedes.

1.3 Pertinent Data

1.3.1 Drainage Areas - 14.5 Square Miles

1.3.2 Discharge at Damsite

Maximum known flood at damsite - 14,000 CFS

Gated spillway capacity at pool elevation - 19,000 CFS

Gated spillway capacity at maximum pool elevation - 20,000 CFS

Total spillway capacity at maximum pool elevation - 20,000 CFS

1.3.3 Dam and Reservoir Data. Petinent data on the dam and reservoir are shown in the following table:

TABLE 1.1 - DAM AND RESERVOIR DATA

ITEM	ELEVATIONS	RESERVOIR AREA	RESERVOIR CAPACITY		LENGTH
				WATERSHED	
	FT MSL	ACRES	AC.FT.	INCHES	MILES
Top of dam	211.5	175	3020	3.9	1.8
Top of Bascule gate	209.1	159	2620	3.4	--
Spillway crest	197.0	85	1170	1.5	--
Top of emergency gates (Recreational Pool)	208.3	154	2500	3.2	1.6
Crest of emergency spillway	204.0	117	1900	2.5	--
30-inch gate valve	144	0	0	0	--
Normal streambed	142	0	0	0	--

SECTION 2 - ENGINEERING DATA

2.1 Design: The original and the 1942 modification designs do not exist. All design records for the remedial work performed after Hurricane Agnes are available through Whitman, Requardt and Associates (Mr. John Gillett), 1304 Saint Paul Street, Baltimore, Maryland 21202.

2.2 Construction: Construction records for the original dam exist in the form of photographs, specifications and contract drawings. All of these are on file with the LBWID (Mr. Stuart Finley).

2.3 Operation: Automatic gate operation is provided by either of two control systems designated "A" and "B". Normal mode of operation is the "A" system. The systems are separate although there are several instruments common to both. System "A" senses the lake level by the autocon bubbler system. Magnetrol float system senses lake level for the "B" system. The control system compares the lake level signal with the gate position signal. If there is a differential between the two signals, the gate hydraulic system will be activated to operate the gate. The gate may be operated manually by the use of hand valves and a gasoline engine driven pump. There is no backup electrical power source.

2.4 Evaluation: While many records are missing from past designs, the LBWID has developed an adequate and valid set of engineering data through the efforts of Whitman, Requardt and Associates. The data are available through the LBWID.

SECTION 3 - VISUAL INSPECTION

3.1 Findings: Information observed in the field is outlined in Appendix C. The visual inspection revealed erosion of the cyclopean masonry on the right downstream core wall. There was no apparent erosion, settlement or sloughing in the earth embankments. Indications of seepage were undetectable due to poor climatic conditions. The force of water from past storms has eroded a pool of undetermined dimensions at the toe of the structure. Upstream of the dam, there is considerable residential development. Immediately downstream from the dam is Virginia Highway 244, (Columbia Pike).

3.2 Evaluation: The visual inspection revealed no apparent problems that would require immediate action.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 Procedures: The water level of Lake Barcroft is maintained at an elevation of 208.5 feet MSL by the hydraulic Bascule gate. Any rise in the lake level above elevation 208.6 will automatically activate the gate, gradually lowering its position to maintain the 208.5 lake elevation. The gate will reach its maximum open position at 197.1 MSL when the water rises to elevation 210. As the lake level drops, the gate will automatically adjust until it is fully closed at lake elevation 208.6.

4.2 Maintenance of Dam: Since the aftermath of Tropical Storm Agnes, the maintenance of the dam has been very good. The dam was fully analyzed and put into operational condition. Regular periodic inspections by Whitman, Requardt and Associates cover all aspects of the dam.

4.3 Maintenance of Operating Facilities: The Lake Barcroft Watershed Improvement District retained Whitman, Requardt and Associates to develop an operating and maintenance manual (Appendix F - Reference 3). Operational and maintenance procedures are well defined for each item of equipment with step by step instructions for manual operations and emergency procedures. A maintenance contract with Honeywell provides for a quarterly inspection of the control equipment and for service calls in the event of a malfunction.

4.4 Description of Warning System: An annunciator alarm system monitors thirteen points in the control system of the dam. A local visual and audible alarm indicates a malfunction of equipment. The audible alarm may be silenced. The visual alarm cannot be reset until the malfunction is corrected. The annunciator will send a signal via telephone lines to the Honeywell surveillance board. The signal will indicate an equipment malfunction, but will not identify the equipment. At least one person from a Barcroft personnel list will be notified by telephone. Security alarms are provided at the fence entrance and the control house. Activation of either security alarm transmits a signal over telephone lines to the Honeywell surveillance board. Honeywell will notify the police, dispatch a private guard, and notify Barcroft personnel. The control house alarm will also activate an audible alarm.

4.5 Evaluation: For the intended purpose of the dam, the method of operation and maintenance are well developed.

SECTION 5 - HYDRAULIC/HYDROLOGIC DESIGN

5.1 Design: The Bascule gate is designed to discharge 19,000 CFS in its fully open position when the pool level is at an elevation of 210.0 feet MSL. The maximum discharge before overtopping the dam at elevation 211.5 is 22,700 CFS.

5.2 Hydraulic Records: There are no official stream gaging stations on Holmes Run. Items automatically recorded include the pool level and gate position.

5.3 Flood Experience: A peak discharge of somewhat less than 14,000 CFS was estimated to have occurred in June 1972 when the right embankment of the dam eroded to a depth of about 40 feet below its normal height. This was the greatest flood runoff which could be recalled or had been documented. However, it does not mean it was the greatest rainfall which had been experienced since earlier conditions of the watershed could tend to reduce the actual flood runoff at that time. Also, the spillway crest was at a relatively low elevation (elevation 204) from 1915 to 1942, thus, providing greater capacity than is now available. Therefore, passage of the flow of 14,000 CFS estimated for June 1972, or larger, would probably have been largely unnoticed if such a flood discharge had occurred prior to 1942.

5.4 Reservoir Operation: Principal control of flows is over the 151 foot-long Bascule gate which revolves over a spillway crest at elevation 197 and with a top at elevation 209.1 when in a closed position. Under normal operating conditions, the Bascule gate is automatically controlled to maintain a water level in the impoundment at about elevation 208.5. The minimum flow of water from the reservoir must be equal to (1) the inflow to the reservoir or (2) the average flow of the stream whichever is least. Since there are no other demands for withdrawal of water from the reservoir, the automatic operation meets these requirements.

The emergency spillway gates can be removed manually after removal of nuts which hold them in place. The probability of these gates being opened in an extreme flood is questionable. Probable floodtime pool levels assume they have not been removed. If removed, the maximum pool level would be lowered by an amount ranging from 0.4 foot in the Probable Maximum Flood to about one foot in floods reaching elevation 209 with the Bascule gate wide open.

Three 20-inch gate valves allow lake water to be drawn into the wet well from various elevations. A 24-inch gate valve designed to discharge water from the wet well into the river is fixed in the open position inoperative and not needed under present operating requirements. A 30-inch gate valve is located at a low elevation in the wet well to discharge water from the lake directly downstream. These two could be used, if necessary, to dewater the reservoir.

Spillway discharge capacity curves were computed and extended to higher elevations to include that discharge over the top of the emergency spillway and non-overflow section. Reservoir elevation area and capacity curves utilized are contained in an earlier report of the project by an engineering firm. An approximate tailwater rating curve was developed by computing a rating on the downstream side of a highway crossing downstream from the dam and determining the drop between the upstream and downstream sides of this road in the Probable Maximum Flood. Definition of the curve in intermediate points was approximated by consideration of the physical data of the highway crossing and flows involved. Reservoir performance in floods was approximated by use of triangular hydrographs of peak flows with appropriate volumes and as developed in the following paragraphs.

5.5 Flood Potential: A peak inflow for the 1 percent exceedence frequency flood determined by the Hydrology Section of the Baltimore District, Corps of Engineers was considered adequate. The peak inflow for the PMF was calculated from Myer's Formula wherein:

$$Q \text{ (CFS)} = 10{,}000 \text{ (Drainage Area)}^{\frac{1}{2}}$$

5.6 Overtopping Potential: The probable rise in the reservoir and other pertinent information on the reservoir performance in various floods is shown in Table 5.1, page 12.

5.7 Reservoir Emptying Potential: Assuming an average inflow of 15 cfs, it would take less than two days to draw down the reservoir from pool level 208.5 to 142.0 feet MSL. This assumes the Bascule gate is operable and fully lowered, the gates are removed from the emergency spillway, the gates on the three 20-inch wet well inlets and the 30-inch outlet are operable and fully opened, and the gate on the 24-inch wet well outlet remains fixed in the open position.

5.8 Evaluation: Strict adherence to the hazard (high) and size (intermediate) classification ascribed to the project indicates a recommended Spillway Design Flood equal to the PMF. Such a flood would overtop the non-overflow section of the dam by 3 to 4 feet. The project is capable of passing a flood of 25,000 CFS, which is larger than both the 100-year and one-half the PMF, without passing water, over the non-overflow sections of the dam. A flood of 25,000 CFS is estimated to have a probability of occurring once every 200 to 500 years on an average or 0.5 to 0.2 percent chance of occurring in any one year.

TABLE 5.1 - RESERVOIR PERFORMANCE

ITEM	AVERAGE FLOW	FLOOD ONE PERCENT 1/	1/2 PMF	PMF 2/
Peak Discharge, cfs:				
Inflow -	15	21000	20000	40000
Outflow -	15	20200	19800	38300
Peak Elevation, FT MSL	208.5	210.1	209.9	214.8
Principal Spillway:				
Depth of Flow, FT	-	13.1	12.9	17.8
Avg. Velocity, FPS	-	10.1	10.1	11.4
Auxiliary Spillway:				
Depth of Flow, FT	0.2	1.8	1.6	6.5
Avg. Velocity, FPS	2.9	4.3	4.0	8.2
Non-overflow Sections:				
Depth of Flow, FT	-	-	-	3.3
Avg. Velocity, FPS	-	-	-	5.8
Tailwater elevation, FT MSL	142+	166+	166+	183+

1/ The 1 Percent Exceedence Frequency Flood has 1 chance in 100 of being exceeded in any given year.

2/ The Probable Maximum Flood is an estimate of flood discharges that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region.

SECTION 6 - DAM STABILITY

6.1 Structural Stability: The hazard classification requires the dam to be designed for the PMF or a height of water to elevation 214.8. Whitman, Requardt and Associates designed the dam for a flow of 20,800 CFS (1/2 PMF) or a height of water to the top of the non-overflow core walls (211.5 MSL). They based their design on the assumption that overtopping flows will erode the right embankment which will act as an emergency spillway or "safety plug". This erosion, similar to the failure during Tropical Storm Agnes, will relieve excess hydrostatic pressures and maintain the integrity of the concrete structure.

Cross-sectional data and a stability analysis, obtained from Whitman, Requardt and Associates,were reviewed. The analysis follows all Corps criteria for structural stability with the exception of uplift factors at the base of the dam. Corps criteria considers 100 percent uplift pressures; whereas, Whitman, Requardt and Associates used 50 percent.

If the right abutment does not wash out, higher levels of water will place larger forces on the dam. Columbia Pike runs parallel to the face of the dam approximately 250 feet downstream. The stream is restricted in its flow by a single arch-shaped culvert under the road. This restriction causes the development of a high tailwater during the PMF to at least elevation 183. A stability check, using information from the analysis by Whitman, Requardt and Associates and the PMF, indicates the dam falls outside Corps criteria. However, it is within acceptable stability limits according to the state-of-the art (Appendix D).

6.2 Foundation: The gravity structure bears on rock. The original construction specifications called for rock to be thoroughly cleaned and treated with dental concrete. Available construction photographs indicate a keyed base. Also, the pictures showed competent exposed rock with near vertical joints.

The specifications further required a cut-off trench on the upstream side for the whole length of the dam and core walls. This cut-off trench was to have near vertical sides. All springs were to be piped, grouted, and carried outside the dam. If it was required by the contracting engineer, grout holes were to be drilled in the foundation and grouted under suitable pressure. There are no construction records to verify that the above specified work was performed.

According to hearsay, the dam, was embedded in approximately 10 feet of material with a toe elevation of 142 feet MSL. At the time of the inspection, the toe of the dam was submerged under an estimated 10-12 feet of water. The elevation of the water was estimated at 155 feet. Whether or not any erosion of rock at the toe has occurred is unknown.

State geologist J. Roy Murphy visually classified nearby rock outcrops as Wissahickan Schist. It is a fine grained quartz-mica Schist with biotite, hornblende and some feldspars. The feldspars indicate weathering on exposed surfaces. The formation exhibits relic sedimentary bedding and texture. There are inclusions of quartz pebbles and cobbles. Rock outcrops on the left downstream side are quite solid. Bedding strikes N 04° E with a 62° NW dip. The rock mass on the right abutment is deeply weathered with evidence of clay seams between bedding planes up to one-quarter inch thick. The same condition exists in joints. Joint sets were measured at N 20° W, 19° SW; N 63° W, vertical; and N 38° W, 60° NE.

Mr. Murphy performed a post inspection literary study of the area geology. He reported that the geology of the area is not a typical Wissahickon Schist, but, instead, the Boulder Gneiss Lithofacies of the Wissahickon as named by G. W. Fisher (1970)(Appendix F - Reference 4). It was named the Sykesville Formation by G. W. Stose (Reference 5) in 1928 and he at that time described it as, "a granitic looking schistose rock with many inclusions, mainly quartz pebbles and garnets." The material grades into a more typical schist east and west of the Barcroft site. Stose classified the rock as a metamorphosed intrusive granite, but later investigators, notably C. A. Hopson (1964) (Reference 6) determined that they were metamorphosed sandy mudstones containing quartz granules, pebbles and rock fragments. The relic textures also proved a sedimentary origin rather than igneous.

The Baltimore District, Corps of Engineers reported that shear tests in similar rock indicated a shear envelope of $\tau = N \tan 33^\circ + 500$ psi.

6.3 <u>Embankments</u>: The dam core walls originally were intended to be keyed into rock. However, hearsay reveals the contractor had to overexcavate due to unexpected "rotten rock". Therefore, both walls tie into the embankments which, in turn, tie into rough competent rock as shown in the construction photos. The specifications called for the embankment to be placed in 12-inch lifts thoroughly wet with a jet of water from a nozzle.

In 1972, the right embankment was breached eroding to the top of rock at an elevation of 172 feet. Also, small portions of the downstream left embankment were eroded due to discharge from a storm drain. Remedial work replaced the right embankment and corrected the left embankment.

The right embankment was designed as a "safety plug" against overturning as explained in Section 6.1. The embankment consists of a micaceous silt (ML) and sand (SM) with an impervious core (Appendix A - Plates 5 and 6). However, review of inplace density tests performed

during construction indicated no distinct impervious core. There were 89 tests performed of which 17 were specifically marked for impervious fill. Fourteen of these 17 tests were performed in a (SM) sand not even tested for Atterberg Limits. The remaining three tests were performed on a (ML) silt with a liquid limit of 25 and a Plastic Index equal to 3.

The common fill was composed of the same materials. Therefore, based on the above information, this office cannot define a specific impervious core.

There is slope protection on the downstream embankment along the core wall (Appendix B - Plate 4). The protection consists of riprap overlying a fabric filter cloth. The original slope protection (Appendix A - Plate 6) placed during the initial remedial work was eroded by overflow from the guide wall. This type of overflow is shown in Plate 4 of Appendix B. To correct the erosion, the slope was cut back from 3H:1V to 2H:1V to about elevation 200 and replaced with the filter cloth and riprap. All records relating to the design and construction of the embankment are available through Whitman, Requardt and Associates (Mr. Henry Janes).

6.4 Evaluation: Whitman, Requardt and Associates modified the dam based on the assumption that the right embankment will act as a "safety plug". A stability check based on no safety plug and the PMF indicates the design is within acceptable stability limits according to the state-of-the-art. If the dam is overtopped, erosion of the right embankment will occur. Overtopping flows are expected to exceed 6.0 feet per second which is more than embankments of this nature can withstand before scouring. The extent and results of this type of failure is undetermined. According to Whitman, Requardt and Associates, the 1972 failure was gradual and the outflow was small in comparison to the stream flow.

SECTION 7 - ASSESSMENT/REMEDIAL MEASURES

7.1 Dam Assessment: Corps criteria requires a spillway design flood equal to 40,000 CFS, the PMF. The spillway is capable of passing a flood of 25,000 CFS without overtopping the non-overflow sections of the dam. Therefore, the spillway is inadequate and overtopping flows are expected to erode the right embankment. However, the spillway capacity is not considered seriously inadequate, because it can pass more than one-half the PMF. Stability checks based on one-half to full PMF indicate the design is within acceptable stability limits according to the state-of-the-art.

The visual inspection reveled no apparent problems that would require immediate action. Whitman, Requardt and Associates have developed an adequate and valid set of engineering data. Their operations and maintenance manual and periodic inspections insure the adequacy of the dam.

7.2 Remedial Measures: There is no immediate need for remedial measures. However, the following actions are suggested and should be initiated within 12 months.

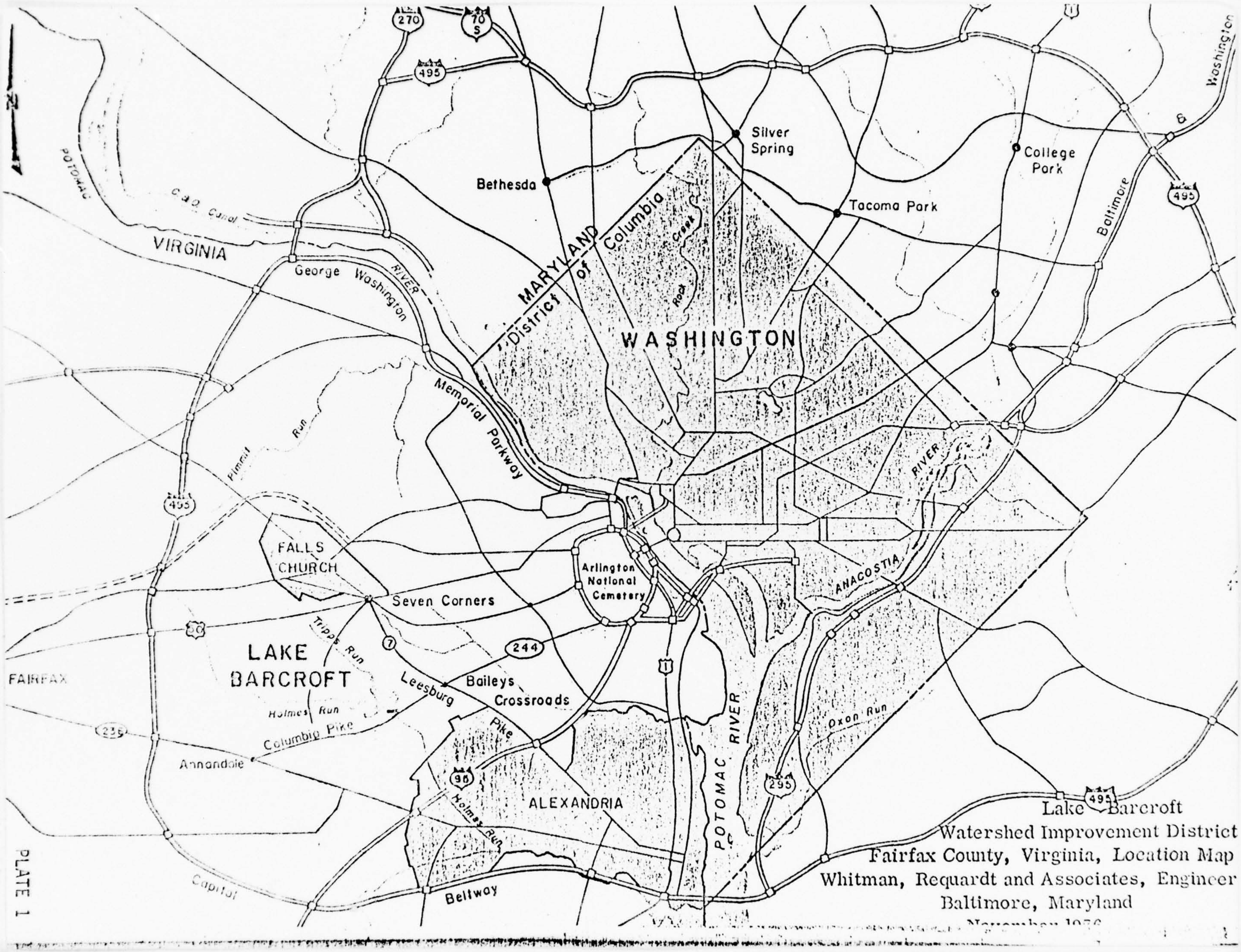
a. Cosmetic repair of the cyclopean masonry surface of the dam to correct spalling and cracking.

b. Inspection of the toe to determine the elevation and condition of foundation material and to correct any problems.

c. Fuses for the control circuits should be made accessible from the front of the control panel. A pilot light should be provided to indicate a blown fuse.

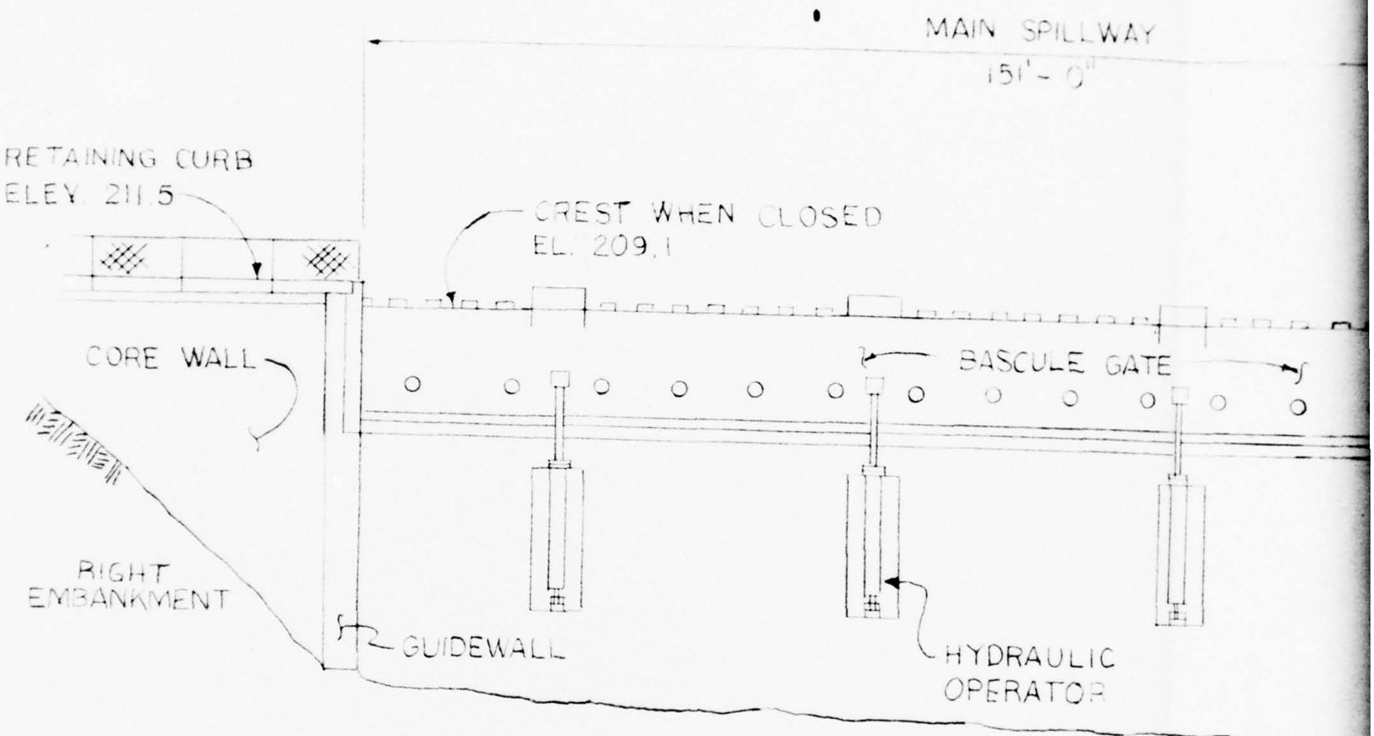
d. Install an emergency light fixture in the control house for operation during a power failure.

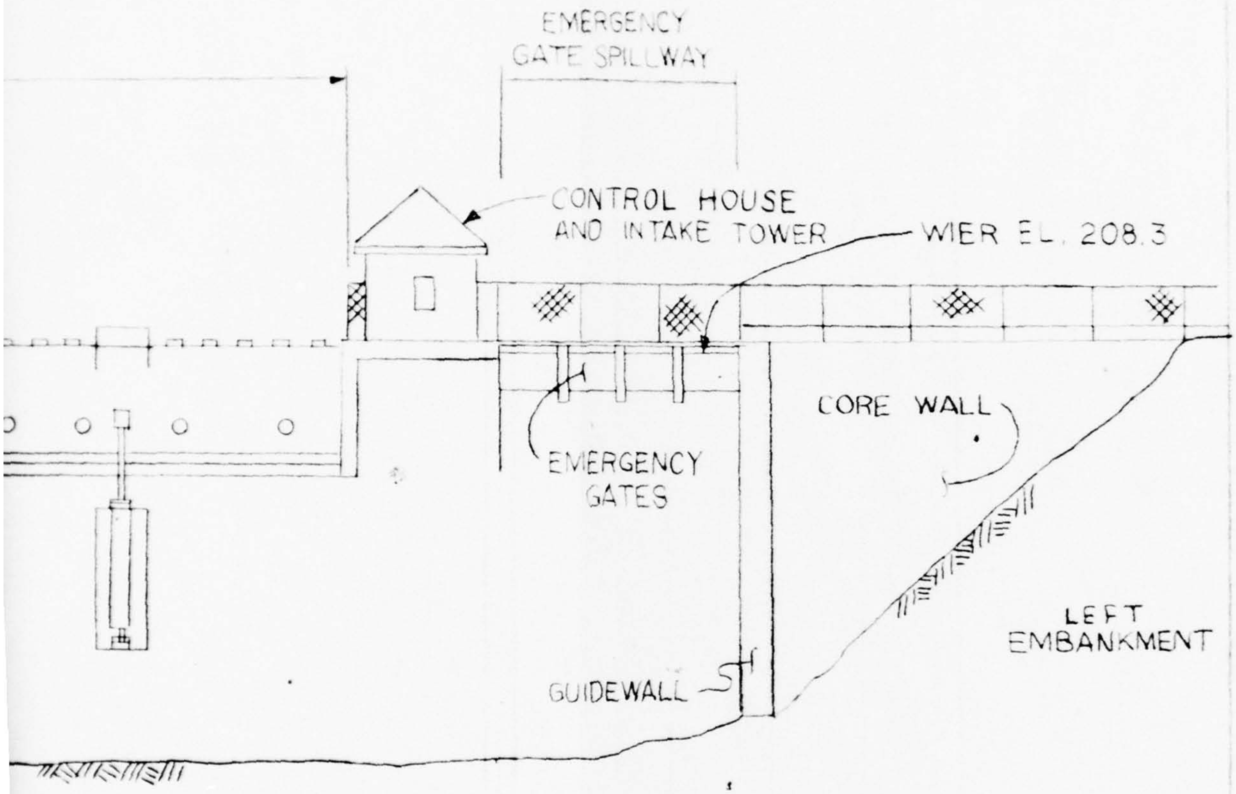
APPENDIX A: MAPS AND DRAWINGS



Lake Barcroft
Watershed Improvement District
Fairfax County, Virginia, Location Map
Whitman, Requardt and Associates, Engineer
Baltimore, Maryland
November 1976

PLATE 1


MAIN SPILLWAY
151'- 0"
RETAINING CURB
ELEV. 211.5
CREST WHEN CLOSED
EL. 209.1
CORE WALL
BASCULE GATE
RIGHT
EMBANKMENT
GUIDEWALL
HYDRAULIC
OPERATOR

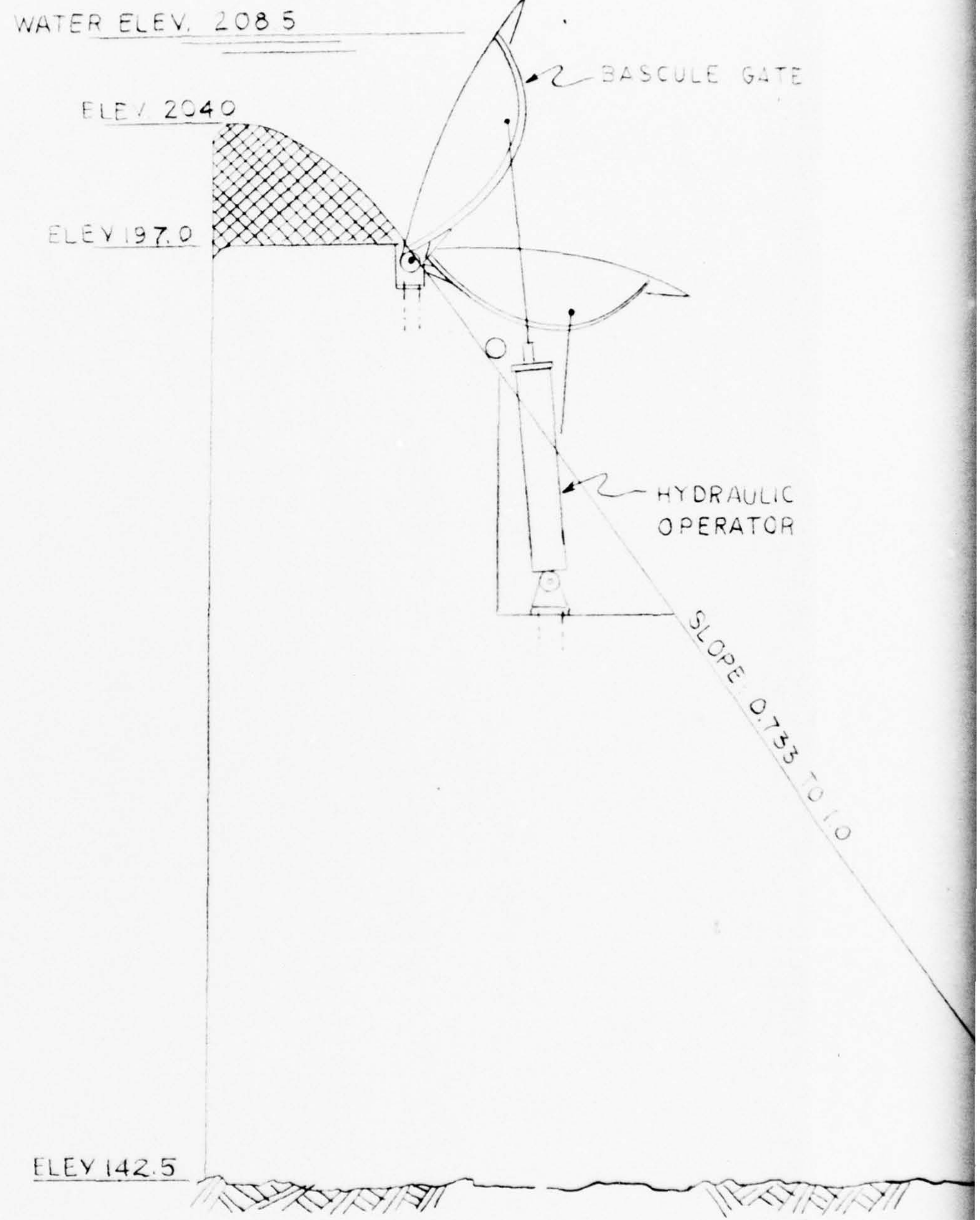


DOWNSTREAM ELEVATION

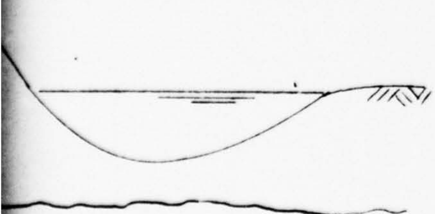
NOT TO SCALE

BARCROFT DAM

PLATE 2



WATER ELEV. 208.5
BASCULE GATE
ELEV 204.0
ELEV 197.0
HYDRAULIC OPERATOR
SLOPE 0.733 TO 1.0
ELEV 142.5



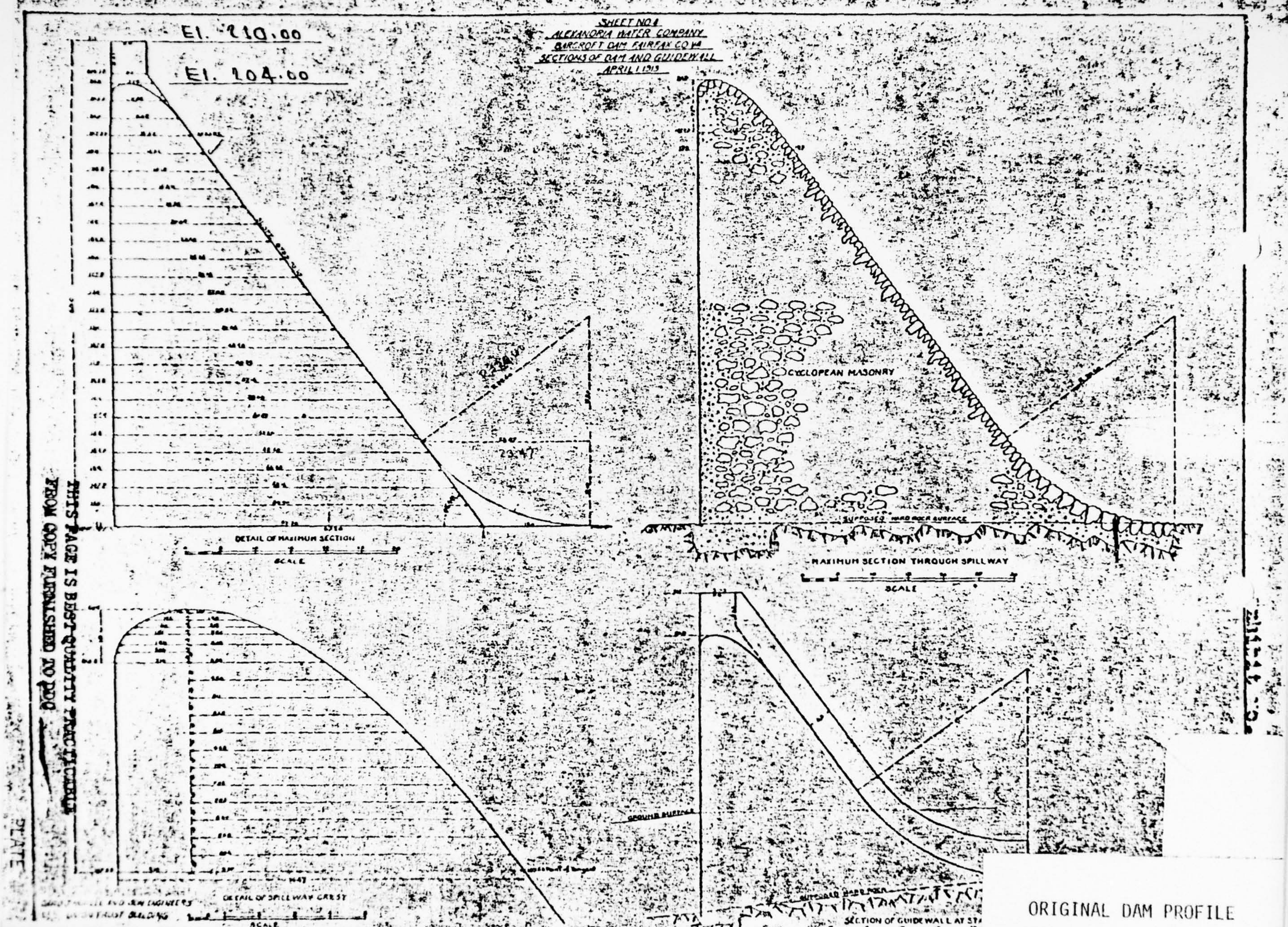
BARCROFT DAM

TYPICAL PROFILE

NOT TO SCALE

2

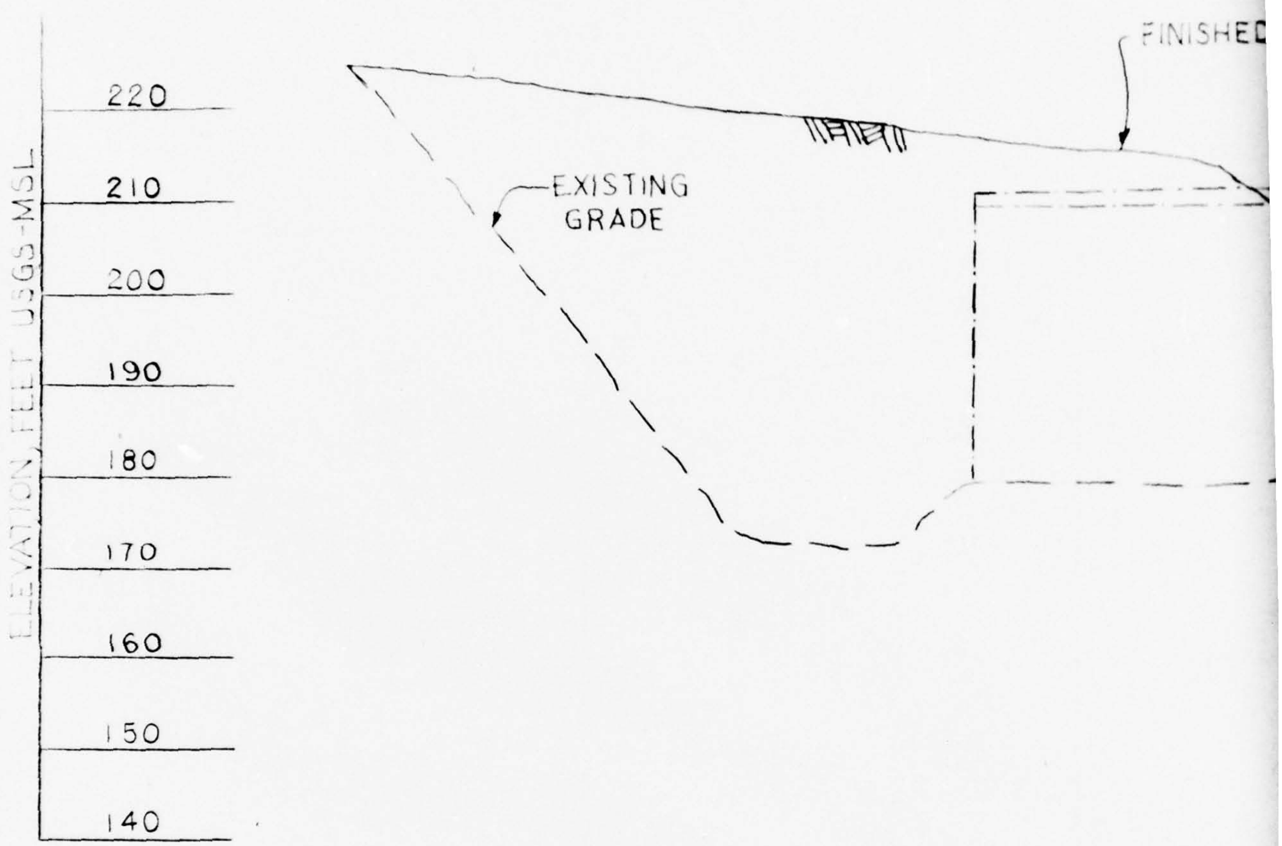
PLATE 3



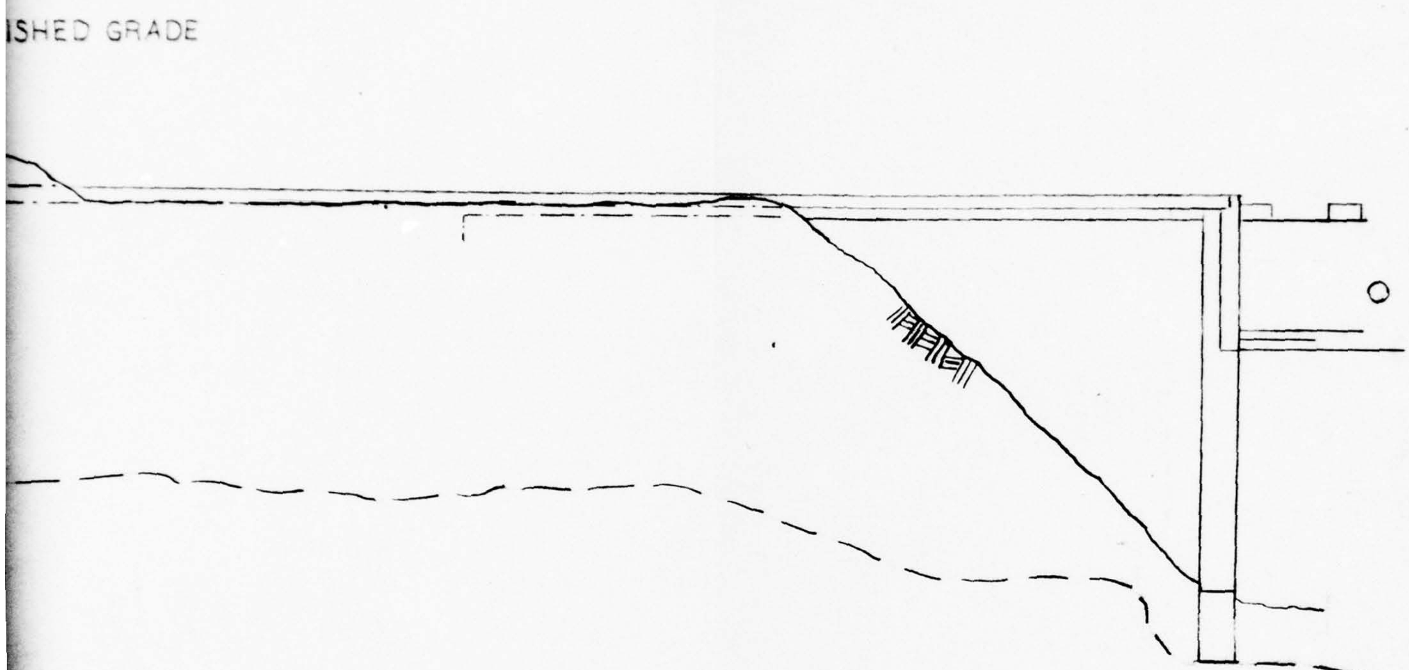
ORIGINAL DAM PROFILE

NOT TO SCALE

PLATE


ELEVATION, FEET USGS-MSL
220
210
200
190
180
170
160
150
140
EXISTING GRADE
FINISHED

1



BARCROFT DAM

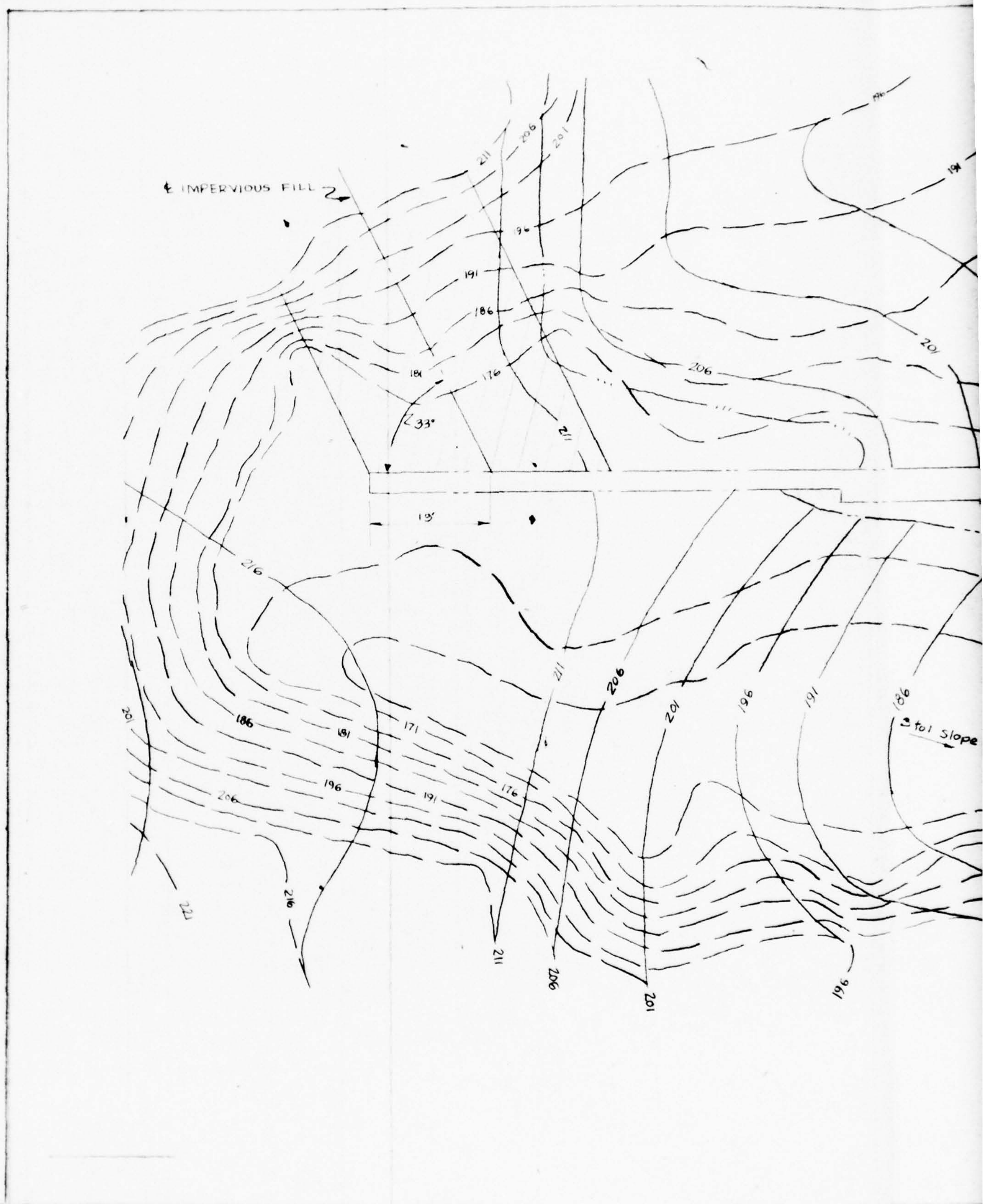
DOWNSTREAM ELEVATION

RIGHT EMBANKMENT

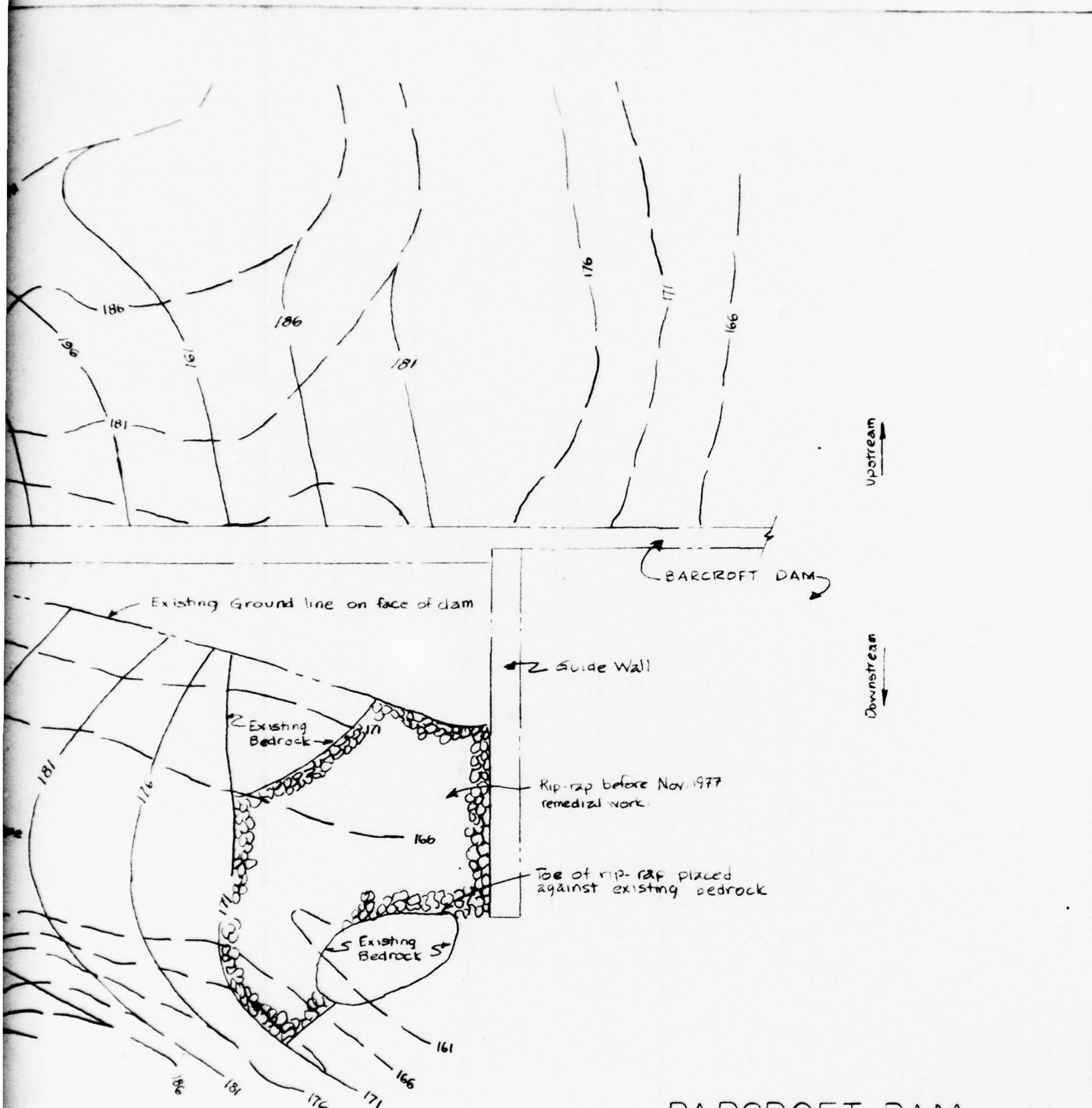
SCALE 1" = 20'

8

PLATE 5



℄ IMPERVIOUS FILL
33°
19'
3 to 1 Slope



BAPCROFT DAM
PLAN VIEW
RIGHT EMBANKMENT
SCALE 1" = 20'

2

PLATE 6

APPENDIX B: PHOTOGRAPHS

BASCULE GATES

DOWNSTREAM FACE OF RIGHT CORE WALL

DOWNSTREAM CULVERT BENEATH VIRGINIA ROUTE 244

PLATE 3

RIGHT ABUTMENT REMEDIAL SLOPE PROTECTION

APPENDIX C: FIELD OBSERVATIONS

FIELD OBSERVATIONS

The visual inspection was conducted on 27 January 1978. It was raining heavily and the temperature was 35° F. There was little or no wind and the ground was covered with several inches of snow. The Bascule gate had been crested and the water was flowing freely down the spillway. Little could be seen of the face except for the non-spillway section of the dam.

CONCRETE STRUCTURE:

On the right downstream core wall, the cyclopean masonry showed many signs of erosion (Appendix B - Plate 2). It had horizontal jointing running across it. One of the vertical expansion joints, installed during the original construction was visible. About 10 feet from the top of the dam, one of the horizontal joints had eroded severely. There was much pock marking and spalling just below that surface for about 5 feet downward. Another 10 to 15 feet down was another joint that had eroded. There were little signs of calcium deposits on the face of the dam. The cyclopean masonry was a very course mix with much aggregate exposed on the face.

FOUNDATION, EMBANKMENTS, AND ABUTMENTS:

The embankments appeared to be a micaceous low plastic (ML) silt. The downstream slopes were covered with grass and were free of heavy vegetation. There was no apparent erosion, settlement, or sloughing. Indications of seepage were undetectable due to the wet weather conditions. Abutment contacts were unobservable. Core wall contacts with the embankment appeared good. There was downstream slope protection covering most of the contact on the right side.

The force of the water from past storms had eroded a pool at the toe. Whitman, Requardt and Associates report that it is approximately 10 to 12 feet deep. Rock outcrops, within the immediate downstream area, indicate a quartz mica schist with relic sedimentary bedding and texture. A rock mass on the right downstream slope was deeply weathered with evidence of clay seams. There were no instruments, wells, or drains on the dam.

REGULATING OUTLET WORKS:

One 30-inch diameter pipe with a gate valve can pass water directly through the dam at a low elevation. Whitman, Requardt and Associates recommend against opening the valve unless absolutely necessary as there is the possibility it could not be closed. The wet well within the intake tower is drained by one 24-inch diameter pipe with a gate valve stuck in the open position. Also, in the tower, there is a control panel for the Bascule gates. The panel is located on top of the hydraulic cabinet. Access to the back of the panel was difficult since maintenance personnel had to climb over the hydraulic cabinet. Working space behind the panel was cramped.

RESERVOIR AND DOWNSTREAM CHANNEL:

Upstream of the dam there is considerable residential development. Approximately 250 feet downstream from the dam is Columbia Pike. The road is approximately a 30-foot embankment with a 60 foot wide arch-shaped concrete culvert passing Holmes Run (Appendix B - Plate 3) beneath the road.

At approximately 1.25 miles downstream from the dam are two homes in the 6200 block of Holmes Run Parkway. These homes are about 250 feet from Holmes Run with their flood elevations 10-15 feet above the streambed. Still further downstream, one to two miles, is a channelization project between Van Dorn and Duke Streets. Cameron Station Military Reservation, containing a Department of Defense Supply Agency is located on the south side of Duke Street where Holmes Run joins Backlick Run to form Cameron Run, approximately 3.5 miles downstream of the dam.

ATTENDEES:

Lake Barcroft Watershed Improvement District:
Dave Alue
Stu Finley
Jack Keith

Whitman, Requardt and Associates:
Jack Gillett
Henry Janes
Parviz Ighani
J. F. Maienshein

State Water Control Board:
Bob Gay
Keith Drohan
Roy Murahy

Corps of Engineers:
Dave Pezza
Mel Cheshire
Lonnie Baird
Jeff Irving
Ken Brooker

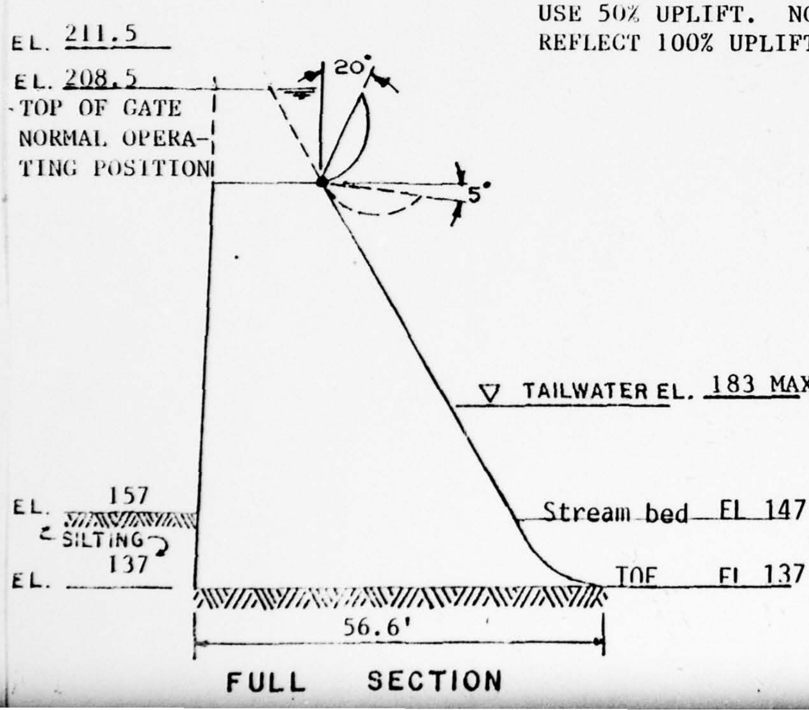
APPENDIX D: STRUCTURAL CALCULATIONS

GRAVITY DAM DESIGN
STABILITY ANALYSIS

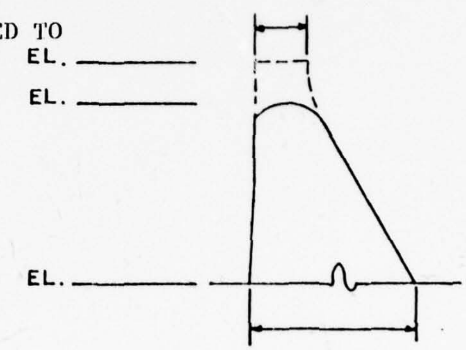
ANALYSIS DONE ON X FULL SECTION ___ PARTIAL SECTION
LOCATION OF SECTION THRU FULL SECTION OF SPILLWAY
ANALYSIS PREPARED BY WHITMAN, REQUARDT & ASSOC., MODIFIED BY J. C. IRVING CORPS OF ENGINEER*

LOADING CASE	ELEV. HEAD WATER	ELEV. TAIL WATER	ΣV	ΣH	$\frac{\Sigma H}{\Sigma V}$	LOCATION RESULTANT FROM TOE	% BASE IN COMPRESSION	FACTOR SAFETY SLIDING	FOUNDATION PRESSURE	
									TOE	HEEL
OPERATING CONDITION	208.5	0	191.9	156.3	0.85	17.22	91	6.58	7.43 KSF	-
1/2 P.M.F	210.0	170	122.5	138.5	1.14	24.1	100	7.0	3.1 KSF	1.19 KSF
P.M.F.	214.8	183	141.3	162.8	1.15	16.5	87	6.1	5.71 KSF	-

* THE COMPUTATIONS BY THE A.&E. USE 50% UPLIFT. NOTES MODIFIED TO REFLECT 100% UPLIFT.



FULL SECTION



PARTIAL SECTION

APPENDIX E: OWNERS

COMMONWEALTH of VIRGINIA

LAKE BARCROFT WATERSHED IMPROVEMENT DISTRICT

NORTHERN VIRGINIA SOIL AND WATER CONSERVATION DISTRICT

LAKE BARCROFT WATERSHED IMPROVEMENT DISTRICT (LBWID)

Trustees:
Leonard A. Alne, Chairman ("Dave")
6234 Lakeview Drive
Falls Church, Virginia 22041
(703) 941-3918

Jack J. Keith, Secretary
3435 Mansfield Road
Falls Church, Virginia 22041
(703) 820-8609

David E. Stahl, Treasurer
3438 Blair Road
Falls Church, Virginia 22041
(703) 671-0193

NORTHERN VIRGINIA SOIL AND WATER CONSERVATION DISTRICT (NVS&WCD)

Directors:
Robert Keating, Chairman
1814 Baldwin Drive
McLean, Virginia 22101
(703) 356-4401

Stuart Finley, Liaison Director to LBWID
3428 Mansfield Road
Falls Church, Virginia 22041
(703) 820-7700

Wayne Smith, Secretary	%NVS&WCD, 3945 Chain Bridge
Patricia Bartz, Vice Chairman	" Road, Fairfax,
Joseph McKinney, Treasurer	" Virginia 22030

BARCROFT BEACH, INCORPORATED (BBI)

President:
Captain Frank M. Sanger
6229 Edgewater Drive OR % P. O. Box 1085, Falls
Falls Church, Virginia 22041 Church, Virginia 22041
(703) 820-1130

BARCROFT LAKE MANAGEMENT ASSOCIATION, INCORPORATED (BARLAMA)
(same as above)

Approximately 80% of the property owners of the Lake Barcroft community (1,000 in number) own a membership share in BARLAMA.

BARLAMA owns BBI.

BBI has contracted with LBWID to operate the dam.

NVS&WCD governs LBWID and appoints WID Trustees.

APPENDIX F: REFERENCES

REFERENCES

1. Department of the Army, Office of the Chief of Engineers. National Program of Inspection of Dams, Volume I, Washington, D. C., May 1975

2. Whitman, Requardt and Associates, Engineers. Study of Long-Range Improvements for Lake Barcroft Dam, Fairfax, Virginia. Baltimore, Maryland, November 1972.

3. Whitman, Requardt and Associates, Consulting Engineers. Lake Barcroft Dam Operation and Maintenance Manual. Baltimore, Maryland, February 1977.

4. Fisher, G. W. The Metamorphosed Sedimentary Rocks along the Potomac River near Washington, D. C., Studies of Appalachian Geology. Central and Southern, Interscience Publishers, 1970, p. 460.

5. Stose, G. W. Geologic Map of Virginia (Scale 1:500,000). Virginia Geologic Survey, 1970, 1928.

6. Hopson, C. A. "The Crystalline Rocks of Howard and Montgomery Counties" The Geology of Howard and Montgonery Counties. Maryland Geologic Survey, 1964, p. 27-215.